DABNEY

Design of a highway bridge

Civil Engineering

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DESIGN OF A HIGHWAY BRIDGE

BY

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This is to certify that the thesis of JOHN BLANTON DABNEY entitled Design of a Highway Bridge is approved by me as meeting this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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DESIGN OF A HIGHWAY BRIDGE.

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DESIGN OF A HIGHWAY BRIDGE.

I. INTRODUCTION.

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1. Location: The site for this proposed bridge is where the county road, between Greenville and Indianola, Mississippi, crosses Bogue Phalia River. It may be more definitely defined as follows: Beginning at the corner of sections five, six, seven and eight of township eighteen, north, and range six, west, in Washington County, Mississippi, thence, east 2256 feet along the road to a bend in same, thence north twenty degrees east 430 feet along the road to a point 100 feet southwest of the west end of the present wooden bridge, thence, north sixty-two degrees east 500 feet along the center line of the present bridge to a point 70 feet northeast of the east end of same.

2. Loading and Specifications: This bridge is to constitute an integral part of the county road between the above named county seats. Ordinary country traffic usually passes over this road, but occasionally it is subject to the traffic of road rollers. Therefore, this bridge shall be designed to withstand such traffic. Ketchum's "General Specifications for Steel Highway Bridges, Class D_1", apply to the above conditions of loading and will, therefore, be used.

In the following pages of this design, the pages in Ketchum's "Design of Highway Bridges" will be referred to by the letter K with the number of the page following it. Thus, (K 51), signifies page 51. The paragraph of the specifications will be desig-

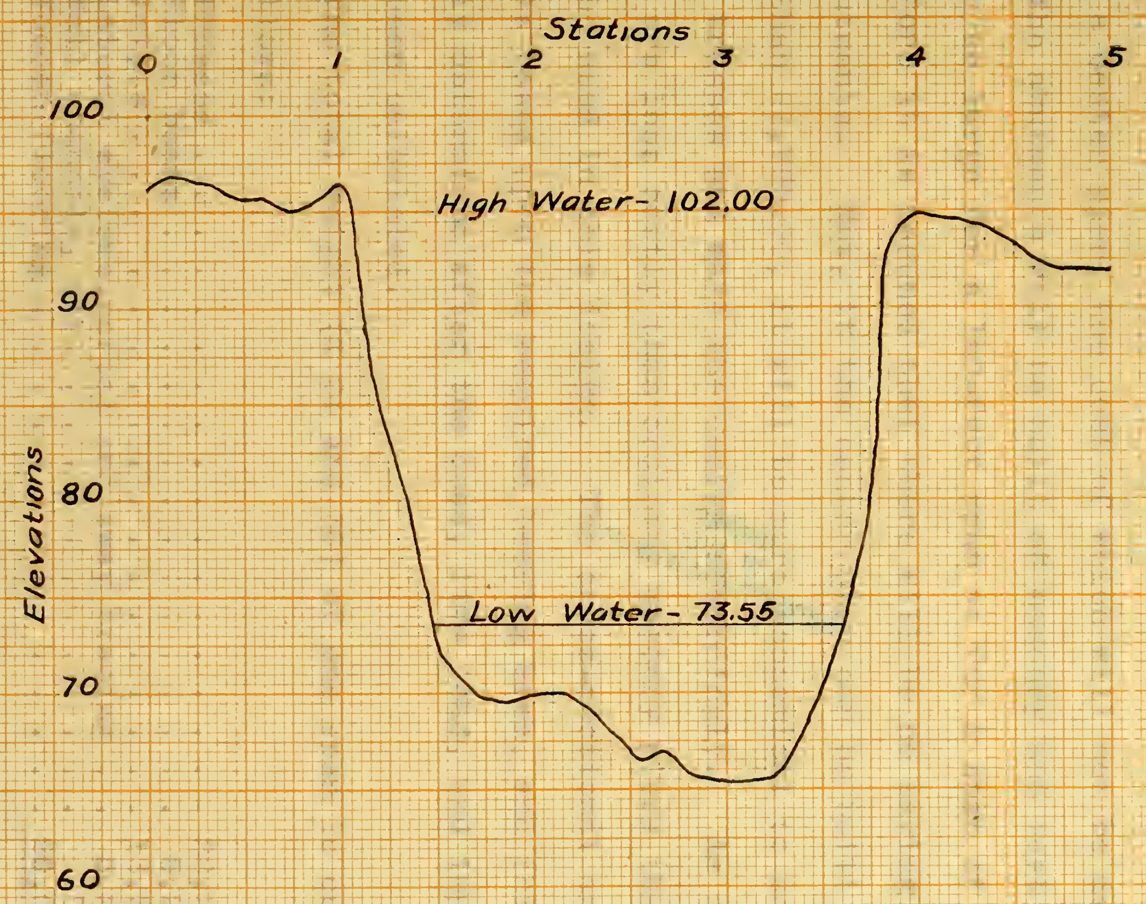
nated thus, (S 15), which signifies paragraph 15 of the specifications. The pages in the Carnegee Handbook will be referred to in the above manner, thus, (C 97) designates page 97 of that book.

3. Length of Bridge: Plate I gives a profile of the stream at the proposed site, and from this the total span between abutments will be determined. The west bank of the stream begins at station 1 + 05, and the east bank is at station 3 + 85, thus making the total distance between banks 280 feet. Therefore, this bridge must be designed to span the 280 feet, and according to (S 7) the distance from center to center of trusses is 15 feet and this will give a clear roadway of approximately 14 feet.

4. Number of Spans: It now becomes necessary to determine the number of spans which will be most economical. According to the American Bridge Company (K 221), steel highway bridges having a span of from 240 to 322 feet should be of the Petit type, and spans of from 80 to 168 feet should be of the parallel chord Pratt type of truss. From the curves (K 27 and 28) it is found, by interpolation, that the weight of two 280-foot Petit trusses, designed for a live loading of 1000 pounds per lineal foot of truss, will weigh 89,200 pounds; and for the same loading, four 140-foot Pratt trusses will weigh 44,600 pounds. As the floor systems, etc. for the same pannel lengths would be approximately the same weight for each type of truss, the economical conclusions may be drawn from the above weights.

The cost of steel made up into highway bridges varies from three to four and a half cents per pound. Taking four cents as an average, the cost of the two Petit trusses would be:

PLATE 1



PROFILE OF BOGUE PHALIA RIVER
AT
CENTER LINE OF BRIDGE

CENTER LINE OF BRIDGE
AT
PROFILE OF BOGUE PHALIA RIVER

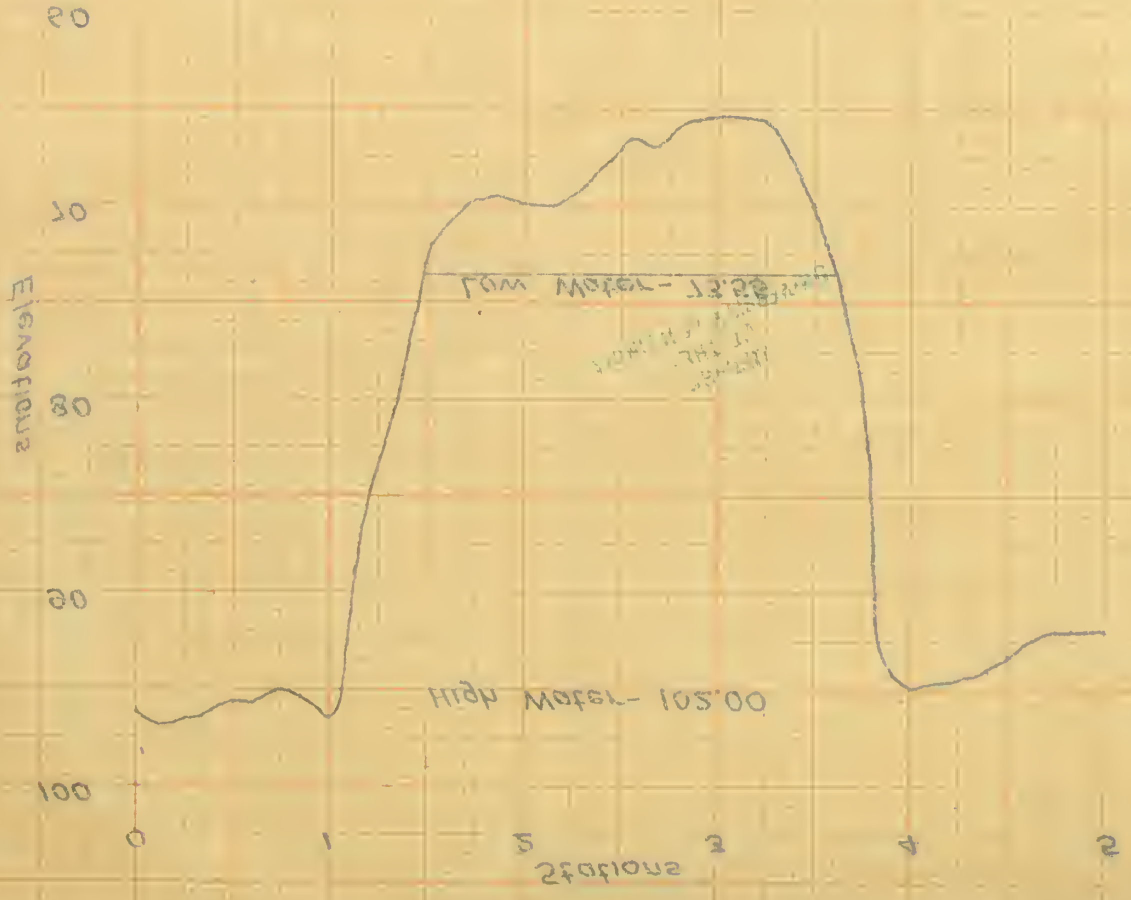


PLATE I

$$\$\ 0.04 \times 89,200 = \$3,570.00,$$

and that of the four Pratt trusses would be

$$\$\ 0.04 \times 44,600 = \$1,780.00.$$

Thus, it is seen that the four Pratt trusses have an economical advantage over the two Petit trusses of

$$\$3,570.00 - \$1,780.00 = \$1,790.00.$$

But in order to use the four Pratt trusses it is necessary to build a center pier, the cost of which will now be determined.

The abutments will be built of masonry and must be approximately as large for a 140-foot span as for a span of 280-feet, and therefore it is assumed that there will be no saving in the cost of abutments. Now, if the center pier can be built for somewhat less than $1,790.00, it will be more economical to select the Pratt trusses and make the design accordingly. As steel tubular piers are less costly than ordinary masonry piers the former type will be used in this design. The cross-sectional area of the pier is based upon the required bearing area, and this may be readily determined after the dead, live load, and impact reactions have been calculated.

The total weight (K 24 and 30) of one span is estimated to be as follows:

Trusses	22,300	pounds
Floor beams	5,200	"
Joists	13,300	"
Lateral systems, portal and sway bracing . .	5,250	"
Wall channels, etc.	550	"
Wooden flooring	29,000	"
Total	75,600	pounds.

This gives a dead load of 540 pounds per linear foot of span.

The end reaction for one truss due to this weight is :

$$R = \frac{75,600}{2 \times 2}$$

$$= 18,900 \text{ pounds.}$$

The weight of steel in this bridge may also be computed by the following formula:

$$w = (24 + 0.82\ L)\ B$$

in which

w = Total weight of bridge per linear foot of span, in pounds,

L = Length of span, in feet, and

B = Clear width of roadway, in feet.

According to this formula the total weight of one span is 79,970 pounds. But as the former eatimate probably more nearly approaches the actual weight of the bridge to be designed, it will be used in the calculation of the dead load stresses.

The total live load (S 20) is:

$$140 \times 14 \times 72 = 141,120 \text{ pounds,}$$

which is 1,008 pounds per linear foot of span, and the live load reaction for one truss is:

$$R = \frac{141,120}{2 \times 2}$$

$$= 35,280 \text{ pounds.}$$

The formula (S 36) for determining the impact stress is :

$$I = S \quad \frac{150}{L + 300}$$

where

I = Impact stress, in pounds,

S = Live load stress, in pounds, and

L = Length of bridge that was loaded in order to produce the maximum stress in the

member, in feet.

Therefore, the impact reaction due to the impact stress is:

$$R = 35\,280 \frac{150}{140 + 300}$$

$$= 12{,}000 \text{ pounds,}$$

and the total reaction for one truss is :

Dead load	=	18,900 pounds
Live load	=	35,280 "
Impact	=	12,000 "
Total	=	66,180 pounds

Now that the end reactions have been determined, the required bearing area on the pier can be calculated. In order to make the required area of the center pier a minimum, the roller ends of these two spans will rest upon the abutments. This will also allow the shoes to be made the same size. According to (S41) the allowable unit bearing stress for concrete is 600 pounds per square inch, and therefore the required bearing area for one truss is

$$A = \frac{66{,}180}{600}$$

$$= 110 \text{ square inches, say one square foot.}$$

The length of the pier should now be approximately determined. On account of overflows or floods, to which the surrounding country is subjected, it is necessary that the centerline of the lower chord be at elevation 103.00. The pedestal will be assumed as being two feet high; and therefore the bearing plate of the pedestal and also the top of the pier will be at elevation 101.00. The bottom of the stream at the pier is at elevation 67.49, but the deepest part of the stream which is only 70 feet

from the pier, is at elevation 63.61. It is reasonable to suppose that the deepest part of the channel shifts from time to time, and at some future date it may be at the center pier. The bottom of the pier should be at least two diameters of the pier below the bottom of the stream to give rigidity to the structure (K 331). It is also necessary that the top of the piling, which must (K 331) extend at least two diameters of the pier up inside the steel tubing, shall be well below the low water line to prevent their decay. Therefore, assuming that the pier will be four feet in diameter, the bottom of it will be at elevation 55.61 say 55.00 and the approximate height of the pier will be 46 feet.

According to (S 5) the members Lo L_1 and L_1 L2 must be made of built up sections, and the net area between the pin and the end of the member must be (S 75) 75 per cent of the net area of the member through the pin. Therefore, a distance of, say, nine inches must be allowed between the center of the pin and the end of the member. There must also be a distance of six inches between the end of L_0 L_1 of one truss and the corresponding end of L_0 L1 of the other truss to permit painting. Thus, the size of the pier, as determined by these conditions, may be determined from the following sketch:

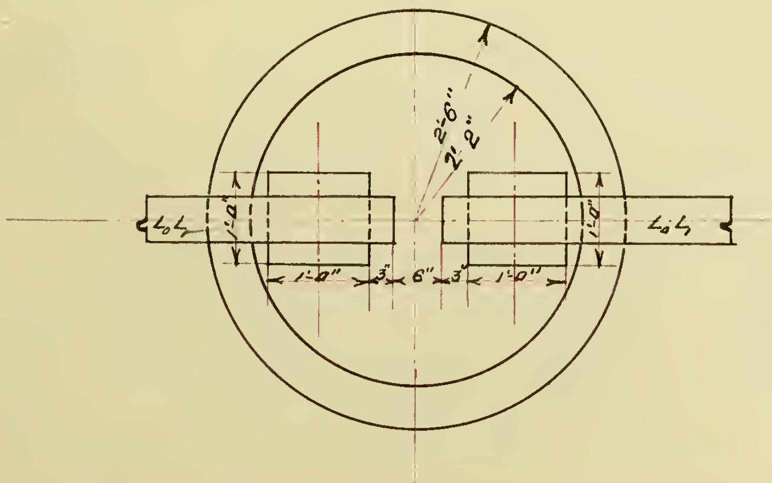


Fig. 1. Dimension of Top of Pier.

Allowing a clearance of three inches between the corners of the bearing plates and the edge of the pier, the diameter of the pier at the top will be 3 feet 6 inches. But according to Cooper's Standards (K 323), the upper portion of the pier for a span of 140 feet must be 4 feet 4 inches, and the lower portion must be 5 feet in diameter. The larger portion must extend up to the low water mark, which in this case is at elevation 73.55, say 74, and therefore the height of the lower tubing is:

$$74 - 55 = 19 \text{ feet.}$$

The thickness of the lower tubing (K 329) must be 7/16 inch, and the thickness of the upper tubing (K 329) must be 3/8 inch. The upper tubing (K 331) must extend its diameter down into the lower portion of the pier. Thus, the total length of the upper tubing is :

$$101.00 - (74.00 + 4.33) = 31.33 \text{ feet.}$$

The weight of a square foot of steel 1/16-inch thick is 17.85 pounds, and a square foot of steel 3/8-inch weighs 15.3 pounds, and therefore the weight of the tubing is :

$$3.1416 \times 4.33 \times 31.33 \times 15.30 = 6{,}520 \text{ pounds}$$

for the upper tubing, and

$$3.1416 \times 5.00 \times 17.85 \times 19.00 = 5{,}330 \text{ pounds}$$

for the lower tubing. This makes 11,850 pounds for all the tubing to which must be added 5 per cent for overlaping, and this makes the total weight of the tubing 12,441 for one pier, or 24,882 pounds for the pair of piers.

The specifications for steel tubular piers (K 329) require the use of diaphragm web bracing between the piers located in streams where floating materials may find lodgment. Therefore, the bracing between these piers shall consist (K 329) of a 5/16-inch

diaphragm web stiffened at intervals of 5 feet with two 6" x $3\frac{1}{2}$" x 3/8" angles running horizontally on each side of the web and bolted rigidly to the diaphragm. This web will begin at elevation 75, which is one foot above the top of the lower tube and extend 25 feet up the pier to elevation 100, which is one foot below the top of the pier. The width of this plate is the distance from center to center of trusses minus twice the radius of the pier, and is

$$15 - (2 \times 2.165) = 10.67 \text{ feet.}$$

As it would be difficult to procure a plate 10'-8" x 5/ 16 x 25'-0" the web will consist of plates 5'-0" x 5/ 16 x 10'-8" riveted together at the stiffeners, the longer legs of the angles serving as connection plates for the diaphragm. The length of the stiffeners must also be 10'-8". The weight of a square foot of steel 5/ 16-inch thick is 12.75 pounds, and therefore the total weight of the web is:

$$25.00 \times 10.67 \times 12.75 = 3400 \text{ pounds.}$$

The weight of two 6" x 3 1/2" x 3/8" angles one foot long is 23.4 pounds, and therefore the total weight of these stiffener angles is:

$$6 \times 10.67 \times 23.4 = 1{,}498 \text{ pounds.}$$

In order to connect the web plate to the pier tubes 3 1/2" x 2 1/2" x 3/8" angles will be placed on each side of the web. This will necessitate the use of four angles, each being 25 feet long and weighing 7.2 pounds per linear foot. The total weight of these connection angles is :

$$4 \times 25 \times 7.2 = 720 \text{ pounds,}$$

and therefore the total weight of steel required for both tubes and bracing is :

Tubing	24,882 pounds
Diaphragm web	3,400 "
Stiffener angles	1,498 "
Connection angles	720 "
Total	30,500 pounds.

Owing to the simplicity of the templates, etc., the shop cost of steel work similar to that in the pier is less than that for steel in bridges and therefore the cost of the steel for the pier will be approximately \$0.035 per pound.

The steel tubes will be filled with Portland cement concrete mixed in the following proportions:

Cement	1 part
Sand	2 1/2 "
Broken stone	5 "

The amount of concrete required to fill the tubes is :

$$\frac{3.1416}{4} \times \frac{2}{4.33} \times 27 = 399 \text{ cubic feet}$$

for the upper tube, and

$$\frac{3.1416}{4} \times \frac{2}{5.00} \times 19 = 373 \text{ cubic feet}$$

for the lower tube. But according to the table (K 326) a tube having a diameter of 5 feet requires 6 piles having a diameter of, say, one foot which must extend two diameters of the pier (K 331) up inside the tube and therefore the cubical contents of the piles must be deducted from the gross amount of concrete required for the lower tube, and thus we have

$$373 - (6 \times 10 \times 0.785) = 326 \text{ cubic feet}$$

of concrete required for the lower tube. This makes a total of 725 cubic feet or 26.8 cubic yards of concrete for one pier, and for the pair of piers 53.6 cubic yards of concrete are required. The cost of concrete in place consisting of the above mentioned

proportions varies from $6.00 to $9.00 per cubic yard, depending on the conditions. On work of this kind $8.00 per cubic yard is a fair estimate.

As stated above, 6 piles will be required under each pier in order to provide sufficient bearing. The total weight on these piles is as follows:

Dead load reaction	18,900 pounds
Live load reaction	35,280 "
Impact	12,000 "
Steel tubing and bracing	15,250 "
Concrete in pier (S 28)	97,500 "
Total . . .	178,930 pounds.

The total weight on each pile is, therefore, 29,830 pounds. According to the following formula from Webb's "Railroad Construction"

$$L = \frac{2\ w\ h}{S + 1}$$ in which

L = Safe load on pile, in pounds

w = Weight of hammer, in pounds

h = Fall of hammer, in feet

s = Set of pile under the last blow, in inches

these piles shall set an average of not more than 2.34 inches for each of the last five consecutive blows from a hammer weighing 2,000 pounds dropped through a height of 25 feet. It will be assumed that the piles must be driven 10 feet into the earth before the above drequirement will be fulfilled, and therefore the total height of each pile will be 20 feet, 10 feet of which extends up into the lower tube (K 331). The total number of feet of piling required for the pair of piers is :

2 x 6 x 20 = 240 feet.

The cost of piles in place varies from $ 0.25 to $ 0.50 per linear

foot. The price in this case will be approximately $ 0.40 per linear foot of pile. The total cost of the pier will therefore be:

30,500 pounds of steel at $ 0.035	=	$1,067.00
53.6 cubic yards of concrete at $8.00 ...	=	429.00
240 feet of piling at $ 0.40	=	96.00
Total	=	$1,592.00

The cost of two 280-foot Petit trusses is $ 3,570.00 and that of four 140-foot Pratt trusses and the cost of the pier is $ 3,372.00. This shows an economical advantage of $198.00 in favor of the four 140-foot Pratt trusses. This difference is sufficient to warrant the construction of the two 140-foot spans with the pier in the center, and the design will be made accordingly.

5. Economical Dimensions: The economical depth for the trusses and the pannel lengths will now be determined. In doing this it should be noted that short pannel lengths give light floor systems and heavy trusses, while long pannel lengths give heavy floor systems and light trusses. For the sake of economy it is not customary to use pannel lengths less than 15 feet for short spans nor more than 25 feet for long spans. According to the American Bridge Company standards (K 221), Pratt trusses with spans of 133 to 147 feet should have seven pannels. This case therefore would necessitate seven pannels of 20 feet each, making a total of 140 feet.

The depth of the trusses is more or less affected by the required head room (S 8). However, in order to have a rigid structure it is necessary that the trusses be deep enough to allow the portal and sway bracing to extend well down the end and intermediate posts. Experience has shown that the most economical conditions occur when the angle ϕ, the tangent of which is the pannel

length divided by the depth, is about 40 degrees. The natural tangent of 40 degrees is 0.839, and letting x equal the required depth of truss, the following equation is true:

$$\frac{20}{x} = 0.839 \text{ from which}$$

$$x = 23.9, \text{ say } 24 \text{ feet.}$$

As this will allow ample head room (S 8) and rigid portal and sway bracing the depth of the trusses will be taken as 24 feet.

II. COMPUTATION OF STRESSES.

- - - - - : - - - - -

Plate II shows an outline of the truss and the lateral, portal and transverse bracings. The length of the end post is :

$$\sqrt{\overline{20}^2 + \overline{24}^2} = 31.25 \text{ feet,}$$

and from this the angle ϕ is computed to be approximately 39° 52', the secant of which is:

$$\frac{31.25}{24} = 1.303$$

The length of the diagonals of the lateral bracing is:

$$\sqrt{\overline{20}^2 + \overline{15}^2} = 25 \text{ feet,}$$

from which the secant of the angle B is found to be :

$$\frac{25}{15} = 1.667.$$

1. Dead Load Stresses.

The total weight of the bridge, as computed on page 10, is 75,600 pounds, or 540 pounds per linear foot of span. This weight is carried by two trusses having seven pannels each, and therefore the dead pannel load is :

$$\frac{75,600}{2 \times 7} = 5,400 \text{ pounds,}$$

from which the dead load reaction for one truss is computed to be :

$$\frac{5,400 \times 6}{2} = 16,200 \text{ pounds.}$$

In computing the dead load stresses in the truss members it is assumed that one-third of the pannel load is acting at the pannel point of the chord top.

The weight of the flooring, guard rail and spiking pieces, which the steel joists and floor beams will be designed to carry, is 15.26 pounds per square foot. These joists will be spaced one foot ten inches or 1.83 feet, center to center, and therefore the uniformly distributed dead load on one joist is :

$$1.83 \times 20 \times 15.26 = 560 \text{ pounds.}$$

This weight, however, does not include the weight of the steel joist. The weight of the joist will be determined later and its weight will be added to the above in order to determine the total dead load moment and shear in the member.

2. Live Load Stresses.

The live load for which the trusses of this bridge will be designed to carry (S 29) is 72 pounds per square foot of clear roadway. The total live load is, therefore,

$$14 \times 140 \times 72 = 141{,}120 \text{ pounds,}$$

or 1,008 pounds per linear foot of span. This weight is carried by two trusses having seven pannels each, and therefore the live pannel load is:

$$\frac{141{,}120}{2 \times 7} = 10{,}080 \text{ pounds,}$$

from which the live load reaction for one truss is computed to be:

$$\frac{10{,}080 \times 6}{2} = 30{,}240 \text{ pounds.}$$

The live load which the floor and its supports will be designed to resist (S 29) is 100 pounds per square foot of clear road way or a 12-ton traction engine with axles 10 feet from center to center and having a gage of 6 feet, two-thirds of the load being carried on the rear axle. The weight on one joist due to the uniformly distributed load of 100 pounds per square foot is, there-

fore,

$$1.83 \times 20 \times 100 = 3,670 \text{ pounds,}$$

from which the end reaction or maximum uniform live load shear (U. l. l. S) is :

$$\frac{3670}{2} = 1,835 \text{ pounds,}$$

and the maximum uniform live load moment (U. l. l. m.) is :

$$\frac{3670 \times 20 \times 12}{8} = 880,800 \text{ pound inches.}$$

3. Wind Load Stresses.

The wind load for the top lateral bracing is 150 pounds per linear foot of span. As this bridge will have transverse bracing, half of this wind load (S 30) is assumed to pass to the lower chord through the sway bracing and intermediate posts, and therefore the wind pannel load (w. p. l.) for the top lateral bracing is :

$$75 \times 20 = 1,500 \text{ pounds,}$$

from which the wind load reaction (w l r) at the upper extremity of the end post is computed to be :

$$\frac{1,500 \times 4}{2} = 3,000 \text{ pounds.}$$

The wind load which the bottom lateral bracing will be designed to resist (S 30) is 300 pounds per linear foot of span, 150 pounds of which is to be treated as a moving load. The moving wind pannel load is, therefore :

$$150 \times 20 = 3,000 \text{ pounds.}$$

The fixed wind load is 150 pounds plus the additional 75 pounds per linear foot transferred from the top lateral bracing, and therefore the total fixed pannel load for the bottom lateral bracing is :

$$225 \times 20 = 4{,}500 \text{ pounds,}$$

from which the wind load reaction is computed to be :

$$\frac{4{,}500 \times 6}{2} = 13{,}500 \text{ pounds.}$$

4. Impact Stresses.

The impact stresses will be computed by the following formula (S 36) :

$$I = S \frac{150}{L + 300}$$

in which

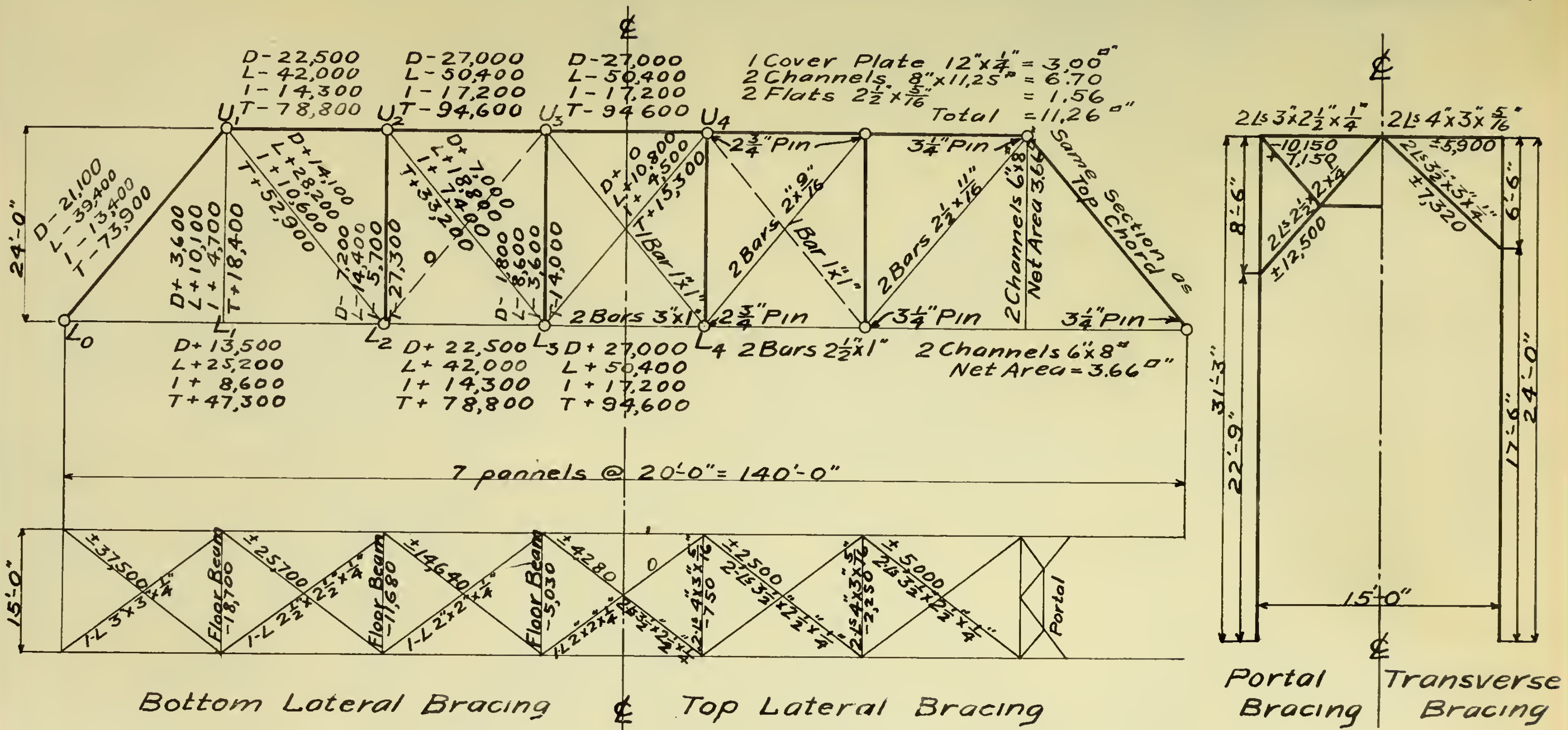
I = Impact stress, in pounds.

S = Maximum live load stress, in pounds, and

L = Length of bridge, in feet, (that is loaded in order to produce the maximum stress in the member.

From the above data the dead, live, wind load and impact stresses will be computed and these stresses will be placed upon their respective members on Plate II. It should be noted that the minus sign denotes compression while the plus sign denotes tension.

PLATE II.



Joists – Spaced 1'-10" C. to C. – 20'-0" Long

DLBM	= 28,100"	D.L.V.	= 585
LLBM	= 264,100	LLV	= 5,000
Impact	= 128,000	Impact	= 2,420
Total	= 420,200	Total	= 8,005

Section Modulus = 420,200 ÷ 16,000 = 26.3
Use – 12" – 31.5# Is – Sec. Modulus = 36.0

Floor Beams – 15'-0" C. to C. Trusses

DLBM	= 241,200	D.L.V.	= 5,100
LLBM	= 694,000	LLV	= 14,680
Impact	= 338,000	Impact	= 7,050
Total	= 1,273,200	Total	= 26,830

Section Modulus = 1,273,200 ÷ 16,000 = 79.6
Use – 18" – 55# I – Sec. Modulus = 88.4

STRESS SHEET OF THE THROUGH PRATT HIGHWAY BRIDGE

DESIGN OF MEMBERS.

The span which is now to be designed is a through-Pratt highway bridge having seven pannels, each of which is 20 feet, making a total span of 140 feet, center to center of end bearings. See Plate II.

Unless otherwise stated, all rivets used in this design will be 3/4-inch in diameter, and all rivet holes 13/16-inch in diameter.

1. The Floor System.

a. The Joists. The joists are 20 feet long, end to end, and they must be composed of steel beams (S 16) having a depth of not less than one-thirtieth of the span (S 19). They will be spaced 1 foot 10 inches, or 1.83 feet, on centers and will be designed to carry (S 29) a uniformly distributed live load of 100 pounds per square foot of total floor surface or a 12-ton traction engine with axles 10 feet from center to center and having a gage of 6 feet, two thirds of the load being carried on the rear axle. The weight on one joist due to the uniformly distributed load of 100 pounds per square foot is :

$$1.83 \times 20 \times 100 = 3{,}670 \text{ pounds,}$$

from which the end reaction or maximum uniform live load shear is:

$$\frac{3{,}670}{2} = 1{,}835 \text{ pounds.}$$

The maximum bending moment due to this loading is, therefore,

$$\frac{3{,}670 \times 20 \times 12}{8} = 101{,}000 \text{ pound-inches.}$$

The impact coefficient (S 36) is :

$$\frac{150}{20 + 300} = 0.47,$$

and therefore the impact shear is :

$$1,835 \times 0.47 = 862 \text{ pounds.}$$

The impact moment due to this loading is :

$$101,000 \times 0.47 = 47,500 \text{ pound-inches.}$$

The above mentioned traction engine, carries 8,000 pounds on each of the rear wheels and 4,000 pounds on each of its front wheels. Each joist, when spaced less than 2 feet on centers, must be designed to carry (S 19) one-half of the concentrated load, or 4,000 pounds of the rear wheel load and 2,000 pounds of the load on the front wheel. The loads and their positions which produce maximum moment is shown in Fig. 2.

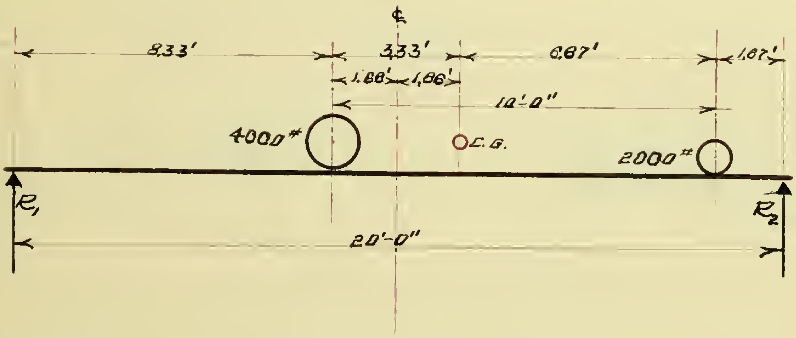


Fig. 2. Position of Traction En ine for Maximum Moment.

The maximum bending moment due to this loading is :

$$\frac{4000 \times 11.67 + 2000 \times 1.67}{20} \times 8.33 \times 12 = 264,100 \text{ pound-inches .}$$

The maximum shear due to the traction engine will occur when the rear wheel is just to the right of the left support and the front wheel is just to the right of the center of the joist. The shear

due to this loading is, therefore,

$$4{,}000 + \frac{2{,}000 \times 10}{20} = 5{,}000 \text{ pounds.}$$

The impact coefficient due to the above loadings is (S 36) :

$$\frac{150}{10 + 300} = 0.484,$$

and therefore the bending moment due to impact is :

$$264{,}100 \times 0.484 = 128{,}000 \text{ pound-inches,}$$

and the shear due to impact for this loading is :

$$5{,}000 \times 0.484 = 2{,}420 \text{ pounds.}$$

It is seen that the moments and shears due to the concentrated live load are larger than those due to the uniformly distributed live load, and therefore these larger values must be used in determining the size of the joists.

The joists will be composed of steel I-beams (S 19), the weight of which was found, by a trial design, to be 31.5 pounds per linear foot. The total weight of one beam is, therefore,

$$31.5 \times 20 = 630 \text{ pounds.}$$

The weight of the flooring is :

$$\frac{29{,}000}{14 \times 140} = 14.8 \text{ pounds per square foot,}$$

and therefore the weight of the flooring carried by one joist is :

$$14.8 \times 1.33 \times 20 = 540 \text{ pounds.}$$

The total maximum dead load moment is :

$$\frac{(630 + 540) \times 20 \times 12}{8} = 28{,}100 \text{ pound-inches,}$$

and the total maximum dead load shear is :

$$\frac{630 + 540}{2} = 585 \text{ pounds.}$$

From the above computations the total bending moment is as follows:

Dead load	28,100	pound-inches
Live load	264,100	" "
Impact	128,000	" "
Total . .	420,200	pound-inches

and the total shear at one end of the beam is found to be :

Dead load	585	pounds
Live load	5900	"
Impact	2420	"
Total .	8005	pounds.

The allowable bending stress (S 39) is 16,000 pounds per square inch and therefore the required section modulus for the joist is :

$$\frac{420,200}{16,000} = 26.3$$

The highest standard I-beam having a section modulus equal to or greater than 26.3 is a 12-inch, 31.5 pound beam (C 97), which is sufficient in strength and therefore it will be used for the joists. The joists adjacent to the trusses will be composed of 12-inch, 20.5 pound channels because the load to which they are subjected is much smaller than that of the other joists.

The joists will be connected to the web of the floor beam by means of connecting angles on both sides. These angles must be short enough to fit between the flanges of the joist, and of such thickness that the length, minus the rivet holes, times the thickness will give sufficient area to resist the end shear of the joist. A length of 8 inches can be satisfactorily used, and as the maximum shear is 8,935 or 4,468 pounds for each angle, and assuming that 2 rivets will be required, the required thickness of the connecting angles is :

$$8 - 2 \times \frac{13}{16} \times t = \frac{8,935}{16,000}$$

$$t = 0.088 \text{ or } \frac{3}{32} \text{-inch}$$

$$= \text{say } 3/8\text{-inch.}$$

The legs of these connecting angles must be wide enough to allow ample room, for driving the rivets and therefore 6" x $3\frac{1}{2}$" x 8" x 3/8" angles will be used.

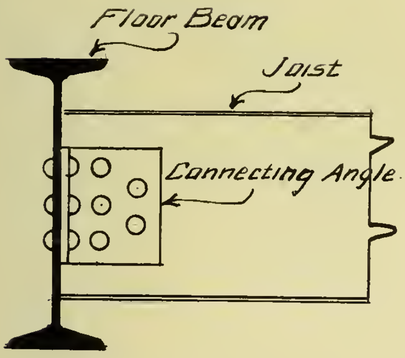


Fig. 3. Connection of Joist to Floor Beam.

The thickness of the joist web is 0.35 inches and by a trial design the connection of the joist to the connection angles was found to be weakest in bearing in the joist web. The allowable bearing stress of a 3/4-inch shop rivet in a 0.35 inch plate is :

$$0.75 \times 0.35 \times 24000 = 6300 \text{ pounds,}$$

and therefore the required number of rivets is :

$$\frac{8,935}{6,300} = 1.42$$

$$= \text{say 4 shop rivets.}$$

By a trial design, the connection of the connecting angles to the web of the floor beam was found to be weakest in shear. The allowable shearing value of 3/4-inch field rivets in single shear is:

Area of rivet x Allowable shearing stress, or

$$0.442 \times 10,000 = 4,420 \text{ pounds,}$$

and therefore the required number of rivets is :

$$\frac{3,935}{4,420} = 2.01$$

$$= \text{say 4 field rivets.}$$

Two of these rivets will be used in the connecting angle on each side of the joist.

b. The Floor-Beam.

The length of the floor beam is assumed to be equal to the distance between trusses, or 15 feet. These beams will also be composed of steel I-beams, the weight of which was found, by a trial design, to be 55 pounds per linear foot. The bending moment due to the weight of the beam is, therefore,

$$\frac{55 \times 15 \times 15 \times 12}{8} = 18{,}600 \text{ pound-inches,}$$

and the shear due to its own weight is :

$$\frac{55 \times 15}{2} = 413 \text{ pounds.}$$

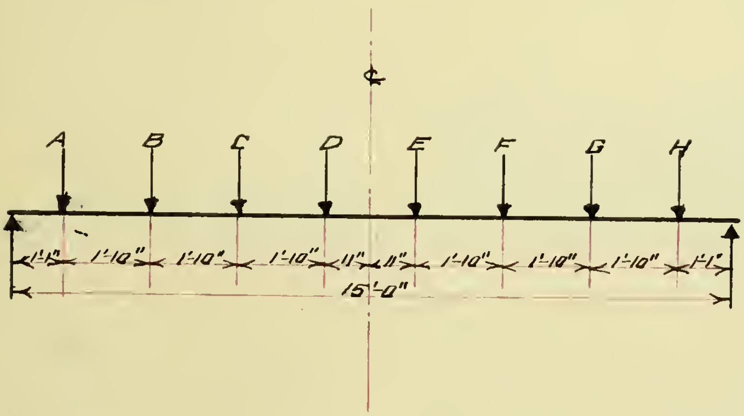


Fig. 4. Positions of Loads on Floor-Beam.

The joist transmits to the floor-beam the weight of the floor and the weight of the joist itself, which are 540 and 630 pounds respectively. The end reaction, or maximum shear, due to this loading is :

$$\frac{540 + 630 \times 8}{2} = 4{,}680 \text{ pounds.}$$

The bending moment in the floor beam due to this loading is :

$4680 \times 79 - 1170 \times 66 - 1170 \times 44 - 1170 \times 22 = 222600$ pound-inches

With a uniform live load of 100 pounds per square foot over the entire road-way, each joist transmits to the floor-beam 3,670 pounds (See page 35). This load is applied at each of the points from A to H, inclusive, in Fig. 4. The end shear in the floor beam due to this loading is :

$$\frac{3,670 \times 8}{2} = 14,680 \text{ pounds,}$$

and the maximum bending moment due to these loads is :

$14680 \times 79 - 3670 \times 66 - 3670 \times 44 - 3670 \times 22 = 675700$ pound-inches

The maximum moment in the floor-beam due to the concentrated live load, or traction engine, is produced when the rear wheels of the engine are placed directly over the floor-beam and the center of gravity of the engine and the center line of the wheels on one side of the engine are placed equal distances on each side of the center line of the floor-beam. Under this condition of loading, a load of :

$$4,000 + \frac{2,000 \times 10}{20} = 5,000 \text{ pounds}$$

will be transmitted to the floor-beam by the joists at the points C, D, F and G, Fig. 4. The left reaction due to these loads is :

$$\frac{5,000 \times (123 + 101 + 57 + 35)}{15 \times 12} = 8,780 \text{ pounds,}$$

and the maximum moment due to these loads is :

$$8,780 \times 79 = 694,000 \text{ pound-inches.}$$

The maximum end shear in the floor-beam due to the concentrated live load is produced when the traction engine is so placed that the joists at the points A, B, D, and E (Fig. 4) will

each transmit to the floor-beam a load of 5,000 pounds, as determined above. Under this condition of loading the shear at the end of the beam is :

$$\frac{5,000\ (167 + 145 + 101 + 79)}{15 \times 12} = 13,750 \text{ pounds.}$$

The impact coefficient (S 36) due to the uniformly distributed load is :

$$\frac{150}{12.8 + 300} = 0.48.$$

The impact shear due to the uniformly distributed load is, therefore,

$$14,680 \times 0.48 = 7,050 \text{ pounds,}$$

and the impact moment due to this load is :

$$675,700 \times 0.48 = 324,000 \text{ pound-inches.}$$

The impact coefficient (S 36) due to the concentrated live load is:

$$\frac{150}{7.3 + 300} = 0.487.$$

The impact shear due to the traction engine is, therefore,

$$13,750 \times 0.487 = 6,700 \text{ pounds,}$$

and the impact moment due to this condition of loading is :

$$694,000 \times 0.487 = 338,000 \text{ pound-inches.}$$

It is seen that the moment due to the concentrated and the shear due to the uniformly distributed live load are larger than the moment and shear due to the uniformly distributed and the concentrated live load, respectively, and therefore the former must be used in determining the size of the floor-beam.

From the above computations the total bending moment is as follows:

Dead load	241,200	pound-inches
Live load	694,000	" "
Impact	338,000	" "
Total .	1273,200	pound-inches;

and the total end shear is :

Dead load	5,100	pounds
Live load	14,680	"
Impact	7,050	"
Total	26,830	pounds.

The section modulus (S 39) required to resist the above moment is :

$$\frac{1,273,200}{16,000} = 79.6$$

The lightest I-beam that has a section modulus equal to or greater than 79.6 is an 18-inch, 55-pound beam, which is sufficient in strength and will be used for the floor-beam.

The floor-beam will be connected to the intermediate post by 6" x $3\frac{1}{2}$" x 3/8" x 1'-2" connection angles placed on each side of the floor-beam web. The size of these angles was determined by the same method as that employed in determining the size of the angles used to connect the joist to the floor-beam. The total end shear in the beam is 26,830; and by a trial investigation it was found that the connection between the angles and the floor-beam is weakest in bearing in the floor-beam web. The thickness of this web is 0.46 inch, and the allowable bearing stress (S 41) of a 3/4-inch shop rivet in this plate is :

$$0.46 \times 0.75 \times 24000 = 8,280 \text{ pounds,}$$

and therefore,

$$\frac{26,830}{8,280} = 3.24$$

$$= \text{say 4 shop rivets}$$

are required to resist the end shear. The required number of rivets to connect the connection angles to the intermediate post

must be determined when the thickness of the channel web of the intermediate post is known.

2. The Trusses.

a. The Pins. The design of the pins requires a simple but lengthy computation. In through Pratt trusses for highway bridges the arrangement of the tension and compression members is usually the same, and therefore it has been possible to design standard pins for given lengths of span and given conditions of loading. Theoretically a different size pin is required at every point, but in practice it is customary to make the pins at L_0, U_1, and L_2 the same size, and all other pins of a different size, usually smaller than those at the above mentioned points. Table XXVIII in Dufour's Bridge Engineering gives the required size of pins for spans of different lengths, from which the pins for this bridge were selected. The sizes of these pins are shown on Plate II. All pin holes will be made 1/16-inch larger than the pin at that point. (S 142, 143).

b. The Tension Members. The tension members will be made of loop-bars or eye-bars, except in the case of L_0, L_2 and $U_1 L_1$, which must be composed of built-up sections (S 5). In the design of tension members in bridges of this kind, experience has proved that bars which have a ratio of thickness to depth of from one-sixth to one-third give the best satisfaction. Therefore the approximate dimensions of bars will be determined by the formula :

$$d = \sqrt{4\ A}$$

where

d = Depth of bar, in inches;

A = Area of bar, in square inches;

and the actual dimensions can be chosen from the market sizes (C 245 to 250).

TABLE I

TENSION MEMBERS

Member	Stresses (in pounds)					Required Area	Number of Bars	Area of One Bar	Section Used	Area of Section Used	Kind of Bar Used
	Dead Load	Live Load	Impact	Total	Unit						
$L_0 L_2$	13,500	25,200	8,600	47,300	16,000	2.96					
$L_1 U_1$	3,600	10,100	4,700	18,400	16,000	1.15					
$L_2 U_1$	14,100	28,200	10,600	52,900	16,000	3.30	2	1.65	$2\frac{1}{2} \times \frac{11}{16}$	3.44	Eye
$L_2 U_3$	0	0	0	0	16,000	0	1	0	1×1	1.00	Loop
$L_3 U_2$	7,000	18,800	7,400	33,200	16,000	2.08	2	1.04	$2 \times \frac{9}{16}$	2.26	Eye
$L_2 L_3$	22,500	42,000	14,300	78,800	16,000	4.93	2	2.47	$2\frac{1}{2} \times 1$	5.00	Eye
$L_3 L_4$	27,000	50,400	17,200	94,600	16,000	5.92	2	2.96	3×1	6.00	Eye
$L_3 U_4$	0	10,800	4,500	15,300	16,000	0.96	1	0.96	1×1	1.00	Loop

Note: All areas in square inches, and all dimensions in inches.

Table I gives the tension members, the dead, the live, the impact and the total stress in the member. It also gives the unit stress (S 37), the required area, number of bars, and the final sizes and kind of bars used.

There is no stress in the member L_2 U_3, but as the member U_2 L_3 is more or less subject to a possible reversal of stress, a 1" x 1" (S 61) loop-bar counter is put in that pannel as a safeguard. Loop bars are used for the members L_2 U_3 and L3 U_4 because the section is so small as to make it difficult to forge an eye at the head of the bar. They are made square instead of having the depth about four times the thickness because when the thickness of an eye-bar or loop-bar head is less than $\frac{1}{2}$-inch, the head is liable to buckle when the load is applied. The loop bars must be made of wrought iron instead of steel because welded steel is not ordinarily considered reliable.

The design of the hip vertical, U_1 L_1, will now be made, and as stated above, it must comply with (S 5) of the Specifications. The total stress in this member is 18,400 pounds, and as the unit stress is (S 37) 16,000 pounds per square inch, the required area is :

$$\frac{18,400}{16,000} = 1.15 \text{ square inches.}$$

As the floor beams will be attached to the intermediate posts and hip vertical above the lower chord, channels large enough to allow this connection must be used. It will be assumed that the section is composed of two 6-inch, 8-pound channels, the gross area of which is 4.76 square inches. Assuming that one rivet hole is taken out of the flange of each channel at any particular section, the net area of the member is 3.66 square inches.

This member will be connected to the upper chord and end post by means of a pin $3\frac{1}{4}$-inches in diameter. According to (S 75) the area through the pin-hole must be 25 per cent in excess of the net area of the member, and back of the pin at least 75 per cent of the net section through the pin. As the net section of the member is 3.66 square inches, the area through the pin hole must be 4.58 square inches or 2.29 for each side and back of the pin must be 3.42 square inches, or 1.71 for each side. The thickness of the channel web is 0.20 inch and the metal removed from it by the pin hole is :

$$0.20 \times 3.30 = 0.66 \text{ square inches.}$$

The gross area of one channel is 2.38 square inches and the net area through the pin is, therefore, 1.72 square inches. The required area through the pin (S 75) is 2.29, thus leaving 0.57 square inches to be supplied by a pin plate. Assuming that the pin plate is 6 inches wide, the required thickness is :

$$\frac{0.57}{6.00 - 3.30} = 0.211 \text{ inches}$$
$$= \text{say } 3/8 \text{ inch.}$$

The total thickness of the channel web and the pin plate is 0.57 inches, and therefore the plength of the member back of the pin is :

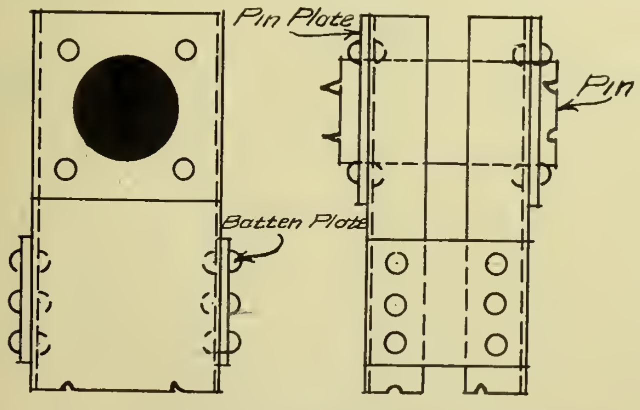


Fig. 5. Connection of High Vertical to Pin at U.

$$\frac{171}{0.57} = 3.00 \text{ inches.}$$

The connection between the pin plate and the channel was found to be weakest in bearing in the channel web. The stress transferred to the 3/8 inch pin plate is :

$$\frac{6,050}{3,600} = 1.68$$

= say 4 shop rivets.

At the lower end, this member is connected to the bottom chord by means of two clip angles and 4 rivets. Only sufficient rivets are required to prevent the sagging of the bottom chord, since the floor-beam will be connected to the hip vertical above the lower chord, and hence no stress comes on the joint at the lower end of the member.

The channels of the hip vertical will be connected together by batten plates (S 69, 61) 6 by $\frac{1}{4}$ inch by 1 foot long, and lattice bars 2 by $\frac{1}{4}$ inches. (S 70, 61).

The member L_0 L_2 must also be composed of a built up section (S 5). The size of this member will depend upon the size of the largest I-bar head at L_2, which is $5\frac{1}{2}$ inches in diameter. In order that the head of the I-bar may have some clearance, the assumed section will be composed of two 6-inch, 8-pound channels. The total stress in this member is 47,300 pounds, and as the allowable unit stress (S 37) is 16,000 pounds per square inch, the required net area is :

$$\frac{47,300}{16,000} = 2.96 \text{ square inches.}$$

The gross area of the above channels is 4.76 square inches, and assuming that one rivet hole is punched in the flanges of each channel at any particular section, the net area of the member is 3.66 square inches. This is somewhat greater than the required area but must be used on account of the size of the I-bar head at L_2 .

Fig. 6 shows the cross-section and general detail of the

member L_0 L_2. The width of the member can not be determined until after the section of the end post is computed, since it must fit inside of the end post, the horizontal legs of the channel being cut off to allow this. By a trial design, the section and general dimensions of the end post was found to be as shown in Fig. 7. Assuming that all of the pin plates are placed on the inside, then the width of L_0 L_2 must be as shown in Fig. 6. This allows a distance of 1/8-inch between the sides of the channels and the end post.

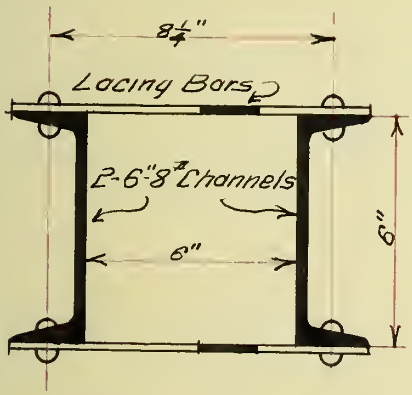


Fig. 6. Cross Section of Lower Chord Member.

The net section of the member L_0 L_2, as determined above is 3.66 square inches, and therefore the net section through the pin hole (S 75) is 4.58 square inches or 2.29 square inches for one side. The depth of the channel is 6 inches and the pin plate will also be assumed to be 6 inches wide, and therefore the required thickness through the pin hole is :

$$\frac{2.29}{6.0 - 3.30} = 0.85 \text{ square inches.}$$

As the thickness of the channel web is 0.2 inch, the required thickness of the pin plate is 0.65 inches, say 11/16 inch. The total thickness of the pin-plate and channel web is 0.89 inch and as the required area back of the pin (S 75) is 1.72 square inches, the required length back of the pin is :

$$\frac{1.72}{0.89} = 1.93 \text{ inches}$$

$$= \text{say } 4 \text{ inches.}$$

Sufficient bearing area must be provided at the point L_o, at which point the diameter of the pin is $3\frac{1}{4}$ inches. The total stress in the member is 47,300 pounds, the total bearing area required (S 41) is :

$$\frac{47,300}{24,000} = 1.97 \text{ square inches,}$$

and the required thickness of one side is :

$$\frac{1.97}{2 \times 3.25} = .304 \text{ inches.}$$

As the above designed pin plate and channel web has a total thickness of 0.89 inches, no additional metal is needed for bearing.

The stress transferred to the 11/16 inch pin plate is :

$$\frac{0.69}{0.89} \times \frac{47,300}{2} = 18,400 \text{ pounds.}$$

The connection between the pin plate and the channel was found to be weakest in bearing in the channel web, and therefore the required number of rivets to connect the pin plate to the channel (S 41) is :

$$\frac{18,400}{3600} = 5.10$$

$$= \text{say 6 shop rivets.}$$

At the point L_2 the legs of the channels may not necessarily be cut off and therefore lighter pin-plates and fewer rivets may be used. However, for economy in the number of tin-plates necessary, the connection at L_2 will be the same as that at L_o.

The channels of this member will be connected together (S 70, 61) by 10 by $\frac{1}{4}$ inch by 1 foot batten plates, and 2 by $\frac{1}{4}$ inch lattice bars (S 70, 61).

c. Top Chord.

In the design of the compression members, the allowable unit stresses are determined by the formula (S 38)

$$P = 16,000 - 70 \frac{l}{r}$$

where

P = Allowable stress, in pounds per square inch.

l = Length of member, in inches.

r = Least radius of gyration, in inches.

According to the Specifications (S 38) the lengths divided by the least radius of gyration, shall not exceed 125 for main members.

The design of the member U_3 U_4 of the top chord will now be made. The length of this member is 20 feet or 240 inches. The assumed section (See Godfrey's Tables page 106) consists of two 8-inch, 11.25-pound channels, and one 12 by $\frac{1}{4}$ inch plate, the total area of which is 9.70 square inches. The above tables also give the eccentricity due to the cover plate as 1.28 inches above the center line of the pin or the neutral axis of the channels. This eccentricity will produce an unequal distribution of stress in the member if the pin is placed in the center of the channels, and therefore small pieces called flats will be placed upon the lower flanges of the channels in order to lower the center of gravity of the section and bring it near the center of the channel webs. Fig. 7 shews a section and the general dimensions of this member. The moment of the cover plate about this eccentric axis is :

$$12 \times \frac{1}{4}(4.00 - 1.28 + 0.125) = 8.53$$

and the moment of the flats about the same axis is :

$$A(4.00 + 1.28 + 0.25) = 5.53\ A,$$

in which A is the area of both flats in square inches. Equating

these two expressions, and solving for A, we get

$$A = \frac{8.53}{5.53}$$

$$= 1.55 \text{ square inches.}$$

The legs of the channels are 2.26 inches wide and assuming that the flats are $2\frac{1}{2}$ inches wide, the thickness of each will be 0.31 say 5/16 inches, thus making the total area of the section 11.26

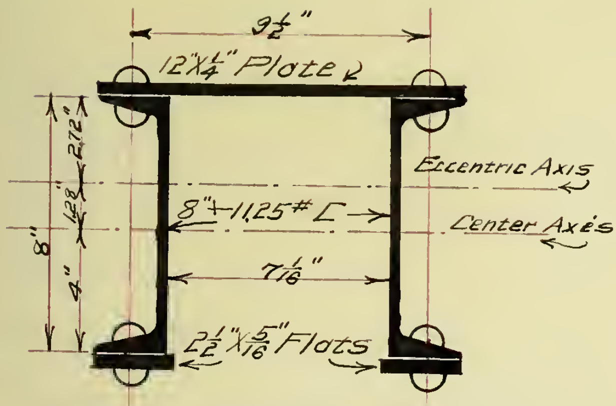


Fig. 7. Section of Top Chord and End Post.

The moment of inertia about the axis through the center of the channels is as follows :

One cover plate	51.1
Two channels	64.6
Two flats	26.1
Total . .	141.8

from which the radius of gyration about this axis is :

$$\sqrt{\frac{141.8}{11.26}} = 3.36 \text{ inches.}$$

Using this value of radius of gyration in the formula, the unit allowable compressive stress is :

$$16{,}000 - 70\,\frac{240}{3.36} = 11{,}000 \text{ pounds per square inch,}$$

and the required area is :

$$\frac{94{,}600}{11{,}000} = 8.60 \text{ square inches.}$$

This area is somewhat less than the actual area of the section but according to (S 61) it must be used for this member.

In order that the section should be safe about both axes, the amount of inertia and radius of gyration about the y-y axis

is 221 and the radius of gyration about the same axis is 4.43 inches. Therefore, since both of these values are larger than those previously computed, it is seen that the section is safer about the y-y axis than about the axis passing through the center of the channels. The y-y axis is perpendicular to the cover plate.

There are small stresses in this member due to its own weight and also to the slight eccentricity of the pins. These stresses seldom exceed 1,000 pounds per square inch on the extreme fiber, and as the actual area of the section is somewhat in excess of the required area, it is evident that the strength of the member is sufficient and therefore these stresses will not be computed.

The above section is for the member in the top chord having the greatest stress. The cover plate is the lightest allowable by the Specifications and in order to use a lighter channel, a change in the depth would be necessitated, and hence cause the joints to be very complicated. Therefore, this section will be used for all sections of the top chord. The sections as finally designed is as follows:

One cover plate	12" x $\frac{1}{4}$"	3.0 square inches
Two channels	8" x 11.25"	6.7 " "
Two flats	$2\frac{1}{2}$" x 5/16"	1.56 " "
	Total	11.26 square inches.

A pin $3\frac{1}{4}$ inches in diameter will be used at the point U_1. The total stress in the member U_1 U_2 is 78,800 or 39,400 pounds for one side, and the bearing area (S 41) required is:

$$\frac{39,400}{24,000} = 1.64 \text{ square inches}$$

for one side. The required thickness is :

$$\frac{1.64}{3.25} = 0.505 \text{ inches,}$$

and since the thickness of the channel web is 0.22 inches, the re-

quired thickness of the hinge plate is 0.285, say 3/8 inch. This makes a total thickness of 0.595 inches. The stress transmitted to the 3/8 inch plate is :

$$\frac{.375}{.595} \times 39,400 = 24,800 \text{ pounds.}$$

The connection between the plate and the channel web is weakest in bearing in the channel web, and therefore the number of rivets required to prevent the plate from rupturing the channel web is :

$$\frac{39,400}{3,960} = 9.95$$

$$= \text{say 10 shop rivets.}$$

Since the ends of the chord members are milled at the splice, and therefore butt up against each other and allow the stress to be transmitted directly, only enough rivets need be placed in the splice to keep the top chord sections in line, say 4 field rivets on each side of the member.

At the point U_2 it is not necessary to put in a pin-plate to take up the stress in the upper chord; but it is only necessary to provide a pin-plate to take up the difference in stress between the two chord sections. The difference in stress is equal to the horizontal component of the maximum stress in $U_2 L_3$. This is 21,200 pounds, and the required area (S 41) for each side is :

$$\frac{21,200}{2 \times 24000} = 0.443 \text{ square inches,}$$

and as the pin used at the point is 23/4 inches in diameter, the required thickness of the bearing area is $\frac{0.443}{2.75} = 0.161$ inches. As this thickness is less than the thickness of the channel web no pin-plate is required.

As the horizontal component of the member $U_3 L_4$ is less than that for $U_2 L_3$, and since the thickness of the channel web at U_3 is the same as that at U_2, it is evident that no pin plate is necessary at U_3.

The under parts of these members must be stiffened by batten plates, which must (S 69) be equal in length to the greatest width of the member. This is 12 inches. Therefore they will be made 12 inches wide and 1 foot long. The thickness of these plates must be (S 67, 61):

$$\frac{9.50}{40} = 0.238$$

$$= \text{say } \tfrac{1}{4} \text{ inch.}$$

The lattice bars shall be (S 70, 71) 2 inches wide, $\frac{1}{4}$ inch thick, and make an angle of 45 degrees with the member.

d. End Post.

Since the direct stress in the end post due to dead, live and impact loads is somewhat less than in the member just designed, the section used for the top chord is assumed to be sufficiently large for the end post, and its efficiency will now be investigated.

Not only is the end post stressed due to the above mentioned direct loads, but it is also stressed by its own weight, by eccentric loading due to the pin being a slight distance from the center of gravity of the section, and by a bending moment, produced by the wind, at the place where the portal joins the end post. These stresses will now be computed, and due allowance will be made in the design for a combination of these stresses to which the member is subjected.

The total length of the end post is 31 feet 3 inches or

375 inches, the total direct stress is 73,900 pounds, and the assumed section, as shown in Fig. 7, is as follows :

One cover plate	12" x $\frac{1}{4}$"	3.0 square inches
Two channels	8" x 11.25#	6.7 " "
Two flats	$2\frac{1}{2}$" x 5/16"	1.56 " "
	Total	11.26 square inches.

In determining the stress in the end post due to its own weight, the entire weight of the member must not be used in computing the bending action, but only that component which is perpendicular to the end post. In this case it is W sin ϕ, in which W is the total weight of the members, and ϕ is the angle subtended between the end post and the hip vertical. The total weight in this case is 1,500 pounds.

The general formula for computing stresses due to bending when the member is also subjected to compression is :

$$S = -\frac{M\,y_1}{I - \frac{P\,l^2}{10\,E}}$$

in which

S = Stress in pounds per square inch in the extreme upper fiber of the member;

M = Exterior moment causing the bending stress;

y_1 = Distance from the neutral axis to the extreme fiber ;

I = Moment of inertia of the section;

P = Direct stress, in pounds;

l = Length of member in inches; and

E = Modulus of elasticity of steel, usually taken as 28,000,000 pounds per square inch.

Substituting in the above formula the proper values, the compression in the upper fiber of the end post due to its own weight is :

$$S = \frac{1/8 \times 1500 \times 375 \times 0.64 \times 4.25}{141.8 - \frac{73,900 \times 375 \times 375}{10 \times 28,000,000}}$$

= - 1,825 pounds per square inch.

The moment due to eccentric loading is equal to the product of the total stress in the member times the distance from the center of gravity of the channels to the center of gravity of the whole section. Substituting these values in the above formula the compression in the extreme lower fiber of the end post is :

$$S = - \frac{73,900 \times 0.53 \times 4.30}{141.8 - \frac{73,900 \times 375 \times 375}{10 \times 28,000,000}}$$

= - 1,580 pounds per square inch.

It must now be determined whether the end post is fixed or hinged, since the bending moment for a fixed column is only one-half of that for one that is hinged. An end post is considered fixed when the product of one-half of the total direct stress times the distance between the channel webs is greater than the wind load acting at U_1 times the length of the end post. In this case the former is :

$$\frac{73,900}{2} \times 7.06 = 261,000$$

and the latter is :

$$2,250 \times 375 = 843,000.$$

Since the latter is greater than the former, the end post is considered hinged. The bending moment at the foot of the portal strut, which is 22.75 feet from L_o, is

$$2,250 \times 22.75 \times 12 = 614,000 \text{ pound-inches.}$$

In computing the stress due to wind, y must be taken as one-half the width of the cover plate, and the moment of inertia must be taken about an axis perpendicular to the cover plate about its cen-

ter. Substituting these values in the above formula the stress in the extreme outer fiber of the cover plate is :

$$S = -\frac{614,000 \times 6}{221 - \frac{73,900 \times 273 \times 273}{10 \times 28,000,000}}$$

$$= -18,300 \text{ pounds per square inch.}$$

The total unit stress may now be computed. It must be remembered that the compressive stress due to the weight of the member is in the upper fiber while that due to the eccentric loading is in the lower fiber, thus tending to counter-balance each other. The stress due to the wind is in the extreme outer fiber of the cover plate, and diminishes to zero at the center of the ppate. The total direct unit stress is :

$$\frac{73,900}{11.26} = 6,250 \text{ pounds per square inch,}$$

and this added to the 18,300 pounds per square inch due to wind gives 24,550 pounds per square inch on the extreme fiber.

The allowable stress in the member is :

$$16,000 - 70\,\frac{375}{3.36} = 8,200 \text{ pounds per square inch.}$$

This is a little less than one third of the actual stress when the wind is taken into account. However, a highway bridge in such a location as the site for this bridge would probably never have a large if any live load during such a storm as would produce the above stress. Furthermore, the above excessive stress is in the extreme fiber of the cover plate only and diminishes rapidly toward the center of the plate. A larger section could be designed, but this would make the joint at U_1 quite complicated. Since the above section is more than sufficient to take care of the dead, live, and impact stresses, and since the floor beam will be con-

nected to the end post and thus tend to fix it and reduce the bending moment to one-half, no change will be made in the assumed section.

The pin at each end of the end post is $3\frac{1}{4}$ inches in diameter and therefore the hinge plates will be the same for each end. The total stress in the member is 73,900 pounds and therefore the required bearing area (S 41) is :

$$\frac{73,900}{24,000} = 3.08 \text{ square inches,}$$

and the thickness is :

$$\frac{3.08}{3.25} = 0.94 \text{ inches for}$$

both sides or 0.47 inches for one side. The thickness of the channel web is 0.22 inches, thus having 0.25 or $\frac{1}{4}$ inch to be supplied by the hinge plate.

The $\frac{1}{4}$-inch hinge plate will take :

$$\frac{0.25}{0.47} \times \frac{73,900}{2} = 19,700 \text{ pounds.}$$

The connection between the hinge-plate and the channel web was found to be weakest in bearing in the channel web, and therefore the required number of rivets is :

$$\frac{19,700}{3,960} = 4.97$$

$$= \text{say 6 shop rivets.}$$

The bottom plates and lattice bars for this member are the same as those for the top chord.

e. Intermediate Posts.

The member $U_2\ L_2$ will now be designed. The length of the member is 24 feet or 288 inches, and the total direct stress is 27,300 pounds. According to (S 38), the length of the member divided by the least radius of gyration, shall not exceed 125 for

main members, and since the length of this member is 288 inches the radius of gyration about an axis perpendicular to the channel web must be equal to or greater than

$$\frac{288}{125} = 2.30.$$

The intermediate post is assumed to consist of two 6-inch, 8-pound channels, the total area of which is 4.76 square inches. The radius of gyration about the axis perpendicular to the channel webs is 2.34, thus satisfying the conditions. The allowable unit stress (S 38) is :

$$16{,}000 - 70\frac{288}{2.34} = 7{,}400 \text{ pounds per square inch,}$$

and the required area is :

$$\frac{27{,}300}{7{,}400} = 3.70 \text{ square inches.}$$

Fig. 8 shows the cross-section and diaphragm of this member.

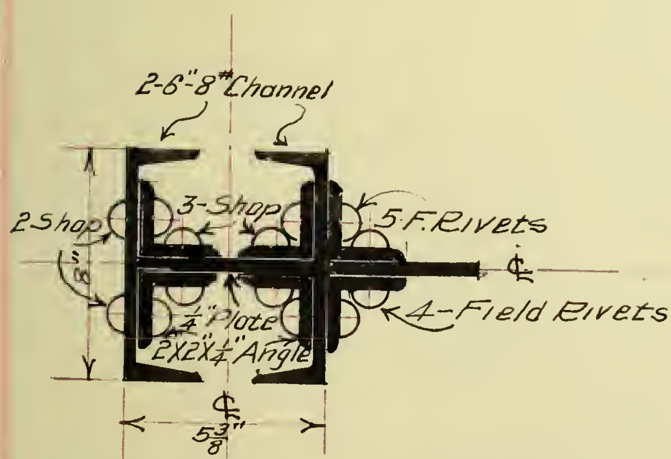


Fig. 8. Cross Section of Intermediate Post Showing Diaphragm.

In order that the intermediate post shall fit up between the channels of the top chord and leave space between it and the top chord for two 9/16-inch I-bars, the distance back to back of the channels must be 5 3/8 inches.

The above radius of gyration was about the axis perpendicular to the channel webs, but in order that the member shall be safe, the radius of gyration about the axis parallel to the

channel webs must be equal to or greater than that about the former axis. It is found)(C 102) that the distance back to back of these channels which will give a radius of gyration equal to the above value is 5.59 inches. Since this is greater than the actual distance, the stability of the member about the axis parallel to the channel webs must be investigated. The radius of gyration about this axis is 1.91. Therefore the allowable unit stress is :

$$16{,}000 - 70\frac{210}{1.91} = 8{,}310 \text{ pounds per square inch,}$$

and the required area is :

$$\frac{27{,}300}{8{,}310} = 3.29 \text{ square inches,}$$

which, being less than 4.76, shows that the member is safe.

The floor beam is connected to the inner channel and in order to transfer one half the stress to the other channel a diaphragm is necessary. This diaphragm will be made of a $\frac{1}{4}$ inch plate connected to the channels by 2 x 2 x $\frac{1}{4}$-inch angles (S 61). The end shear in the floor beam is 26,830 pounds, and since the connection between the floor beam and the channel is weakest in bearing in the channel web, the required number of rivets is :

$$\frac{26{,}830}{3{,}000} = 8.95$$

$$= \text{say 10 field rivets.}$$

The connection between the diaphragm and the angles is weakest in bearing in the diaphragm and therefore the required number of rivets is :

$$\frac{13{,}420}{4{,}500} = 3 \text{ shop rivets.}$$

The connection of the angles to the outer channel is weakest in bearing in the channel web and therefore the required number of rivets is :

$$\frac{13,420}{3,600} = 3.73$$

$$= \text{say 4 shop rivets,}$$

two of which will be used in each angle.

This member is connected to the top chord by a pin 2 3/4 inches in diameter. Since the total stress in the member is 27,300 pounds, the required bearing area (S 41) is :

$$\frac{27,300}{24,000} = 1.12 \text{ square inches,}$$

and the required thickness is :

$$\frac{1.12}{2.75} = 0.4 \text{ inch,}$$

or 0.2 inch for each side. The thickness of the channel web is 0.2 inch and therefore no pin plate is necessary. Since the pin at the lower end is $3\frac{1}{4}$ inches in diameter, it is evident that a less thickness is required and therefore no pin plate is necessary at that end of the member.

The channels of this member will be connected together by $5\frac{1}{2} \times \frac{1}{4} \times 8$-inch batten plates (S 69, 61), and by $2 \times \frac{1}{4}$-inch lattice bars, which shall make an angle of 45 degrees with the member (S 67, 61).

Since the stress in $U_3 L_3$ is less than that in $U_2 L_2$ a smaller section could be used. However, in order to simplify the design and reduce the required number of tinplates, as well as to have channels of sufficient size to which to connect the floor beam, the member $U_3 L_3$ will be the same as $U_2 L_2$.

3. The Portal.

The size of the portal opening was determined by the required clearance (S 8) and is as shown in Plate II . The portal strut or diagonal will be designed first. The total length is

11.33 feet or 136 inches, and since the relation between the length and least radius of gyration shall not exceed 125 (S 38), the mimimum allowable radius of gyration is :

$$\frac{136}{125} = 1.09$$

for one axis and

$$\frac{136}{2 \times 125} = 0.55$$

for the other axis.

It is assumed that the section is composed of two angles $2\frac{1}{2}$ by 2 by $\frac{1}{4}$ inch, having a gross area of 2.12 square inches (C 115) These angles will have their shorter legs parallel to each other and will be separated by 3/8-inch plates or fillers. The radius of gyration about the axis parallel to the shorter legs is 1.25, and that about the axis parallel to the longer legs is 0.59. The allowable unit stress (S 38) is :

$$16,000 - 70\frac{136}{1.25} = 8,400 \text{ pounds per square inch}$$

for one condition and

$$16,000 - 70\ \frac{78}{0.59} = 6,750 \text{ pounds per square inch}$$

for the other condition of radii of gyration. The stress in the member is 12,500 pounds and therefore the required area is :

$$\frac{12,500}{6,750} = 1.85 \text{ square inches.}$$

Since the area of the section is 2.12 square inches it is amply strong to resist the compression stress.

These angles will now be examined for tension. This stress is also 12,500 pounds. The gross area of the angles is 2.12 square inches, and assuming that one rivet hole is taken out of each angle at any particular section, the net area is 1.71 square

inches, The required area (S 37) is:

$$\frac{12,500}{16,000} = 0.783 \text{ square inches,}$$

which shows that the assumed section is sufficient in strength.

This member will be connected to 3/8-inch connection plates at each end (S 61). The connection is found to be weakest in bearing in the angle and therefore the required number of rivets (S 41) is :

$$\frac{12,500}{3,750} = 3.34$$

$$= \text{say 4 field rivets.}$$

As above determined, four field rivets will be used to connect the portal strut to the end post, and three shop rivets will be used to connect it to the top piece.

The top strut of the portal, which has a compression stress of 10,150 pounds, will also be composed of two angles. The total length of the member is 15 feet or 180 inches. The minimum allowable radius of gyration (S 38) is :

$$\frac{180}{125} = 1.44$$

about one axis, and

$$\frac{180}{2 \times 125} = 0.72$$

about the other.

The section is assumed to consist of two 3 by $2\frac{1}{2}$ by $\frac{1}{4}$ inch angles spaced 3/8 inches apart and having their shorter legs parallel, the gross area of the section being 2.62 square inches (C 115). The radius of gyration of this section about the axis parallel to the shorter legs is 1.45 and that about an axis parallel to the longer legs is 0.75. The allowable compressive stress (S 38) is :

$$16{,}000 - 70\frac{180}{1.45} = 7{,}200 \text{ pounds per square inch}$$

for one condition, and

$$16{,}000 - 70\ \frac{.90}{0.75} = 7{,}600 \text{ pounds per square inch}$$

for the other condition of the slenderness ratio. The required area is :

$$\frac{10{,}150}{7{,}200} = 1.41 \text{ square inches,}$$

and since this is less than the actual area the member has sufficient strength to resist compression.

The member will now be examined for the tensile stress of 7,150 pounds. The gross area is 2.62 square inches, and assuming that one rivet hole is taken out of each angle at any particular section the net area is 2.21. The required area (S 37) is :

$$\frac{7{,}150}{10{,}000} = 0.447 \text{ square inches.}$$

It is seen that the required area is somewhat less than the actual area, but on account of the limiting values of the slenderness ratio this section must be used.

The ends of this member will also be connected to the end post by means of 3/8-inch connection plates (S 61). This connection was found to be weakest in bearing in the angles and therefore the required number of rivets is :

$$\frac{10{,}150}{3{,}750} = 2.71$$

$$= \text{say 3 field rivets.}$$

Those members of the portal which do not take any stress will be made of 3 by 3 by $\frac{1}{4}$ inch angles (S 61), and will have three shop rivets in their connections.

4. The Transverse Bracing.

The transverse bracing will be of the same type as that used for the portal except that the top member will consist of two angles spaced at a distance apart equal to the depth of the top chord, and they will be joined together by lacing bars.

The top member will be designed first. The length of the member is 180 inches, and the limiting value of slenderness ratio is 150. Therefore the minimum allowable radius of gyration is :

$$\frac{180}{150} = 1.2.$$

The tensile and compressive stress is each 5,900 pounds, and since only the top angle is supposed to carry stress, the section is assumed to consist of two 4 by 3 by 5/16 inch angles, each having an area of 2.09 square inches. The shorter leg of the angle will be placed vertically; and since the radius of gyration about an axis parallel to the shorter leg is 1.27 the unit stress (S 38) is :

$$16{,}000 - 70\,\frac{180}{1.27} = 6{,}100 \text{ pounds per square inch.}$$

The required area is :

$$\frac{5{,}900}{6{,}100} = 0.968 \text{ square inches.}$$

This is much less than the actual area but must be used on account of the slenderness ratio. Since this angle will be joined to a 3/8-inch connection plate (S 61) the connection is weakest in shear and therefore the required number of rivets (S 40) is :

$$\frac{5{,}900}{4{,}420} = 1.34$$

$$= \text{say 3 field rivets.}$$

According to (S 61,70) the lattice bars will be 1 3/4 by $\frac{1}{4}$ inch, and

make an angle of 45 degrees with the member.

5. Top Lateral System.

According to (S 6) of the Specifications the diagonals of the lateral system will be made of shapes capable of resisting both tension and compression. When shapes are used for these diagonals it is assumed that one half of the stress in the pannel is taken by each diagonal; that is, one diagonal is in tension while the other is in compression. The allowable slenderness ratio for the lateral system (S 38) is 150, and since the diagonals are 25 feet long, the minimum radius of gyration in one direction is :

$$\frac{25 \times 12}{150} = 2.0.$$

As the diagonals cross and are connected at the center of the pannel, the minimum value in the other direction is :

$$\frac{25 \times 12}{2 \times 150} = 1.0.$$

Since there are no angles whose radius of gyration is equal to 2, the section must be composed of two angles spaced a distance apart equal to the depth of the top chord. The radius of gyration is taken about the vertical axis if one half of the length is taken; while if the whole length is considered the radius of gyration must be taken about the horizontal axis. The radius of gyration about the latter axis is several times that about the former axis, and therefore the member is amply safe about the horizontal axis if it is safe about the vertical axis.

The section is assumed to consist of two $3\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{1}{4}$ inch angles, the area of each being 1.44 square inches, and the radius of gyration about the vertical axis is 1.12 (C 115). The

allowable unit stress (S 38) is :

$$16,000 - 70 \frac{150}{1.12} = 6,640 \text{ pounds per square inch,}$$

and since the total stress in the first pannel is 5,000 or 2,500 pounds for each diagonal, the required area is :

$$\frac{2,500}{6,640} = 0.373 \text{ square inches.}$$

The required area is very much smaller than the actual area, but since the above angles are the smallest that will satisfy the slenderness ratio condition, they must be used.

This member, the top lateral diagonal, will now be examined for tension, The gross area of one angle is 1.44 square inches, and assuming that one rivet hole is punched in the section the net area is found to be 1.23 square inches. Since the tensile stress in the member is 2,500 pounds, the required area is :

$$\frac{2,500}{16,000} = 0.157 \text{ square inches,}$$

which shows that the member is sufficiently strong.

This member will be connected to 3/8-inch connection plates (S 61), and since the connection is weakest in bearing in the angle, the required number of rivets (S 41) is :

$$\frac{2,500}{3,750} = 0.67$$

$$= \text{say 3 field rivets.}$$

The above angles will be connected together by 1 3/4 by $\frac{1}{4}$-inch lattice bars (S 67, 61).

Since the above member has a largest stress of any diagonal in the top lateral system, and since it is composed of the smallest standard angles which will satisfy the slenderness ratio condition, the above section must be used for all of the diagonals in the top lateral system.

One half inch rivets shall be used in all lattice bars heretofore designed (S 64).

6. The Bottom Lateral System.

The floor-beams will serve as the bottom lateral struts, while the diagonals, as in the above design, must be composed of angles capable of resisting tension and compression (S 6). These diagonals will be attached to the bottom flange of the joists at their intersections, and therefore the unsupported length is 3.33 feet or 40 inches. The total stress in the first panel is 37,500 or 18,800 pounds for each member. The assumed section will consist of one 3 by 3 by $\frac{1}{4}$-inch angle having an area of 1.44 square inches and a radius of gyration of 0.93. The allowable unit stress (S 33) is :

$$16,000 - 70 \frac{40}{0.93} = 13,000 \text{ pounds per square inch,}$$

and the required area is :

$$\frac{18,800}{13,000} = 1.44 \text{ square inches.}$$

This is the exact area of the assumed angle and therefore it will be used.

This angle will now be investigated for tension. The gross area is 1.44 square inches, and since it is assumed that one rivet hole is taken out of any given section, the unit area is 1.23 square inches. The required area (S 37) is :

$$\frac{18,800}{16,000} = 1.18 \text{ square inches,}$$

which shows that the angle is sufficiently strong.

These angles will be connected to the floor-beams, and since the connection is weakest in bearing in the angle, the re-

quired number of rivets (S 41) is :

$$\frac{18,800}{3,750} = 4.7$$

$$= \text{say 5 field rivets.}$$

The total stress in the second pannel is 25,700 or 12,800 pounds for each member. The assumed section consists of one $2\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{1}{4}$ inch angle having an area of 1.19 and a radius of gyration of 0.77 (C 115). The allowable unit stress (S 38) is :

$$16,000 - 70\ \frac{40}{0.77} = 12,360 \text{ pounds per square inch,}$$

and therefore the required area is :

$$\frac{12,800}{12,360} = 1.04 \text{ square inches.}$$

As this is slightly less than the actual area, the assumed angle will be used for the members in the second pannel.

The gross area of the above angle is 1.19 square inches, and since one rivet hole will be punched in the section, the net area is 0.98 square inches. The required area to resist tension (S 37) is :

$$\frac{12,800}{16,000} = 0.80 \text{ square inches,}$$

which shows that the member is sufficiently strong.

This angle will also be connected to the floor-beam, and since the connection is weakest in bearing in the angle, the required number of rivets (S 41) is :

$$\frac{12,800}{3,750} = 3.42$$

$$= \text{say 4 field rivets.}$$

The total stress in the third pannel is 14,640 or 7,320 pounds for each member. The assumed section will consist of one

2 by 2 by $\frac{1}{4}$ inch angle having an area of 0.94 square inches and a radius of gyration of 0.61 (C 115). The allowable unit stress (S 38) is :

$$16{,}000 - 70\frac{40}{0.61} = 11{,}810 \text{ pounds per square inch,}$$

and therefore the required area is :

$$\frac{7{,}320}{11{,}810} = 0.62 \text{ square inches.}$$

This area is somewhat smaller than that of the assumed angle, but since this is the smallest angle allowed by the Specifications (S 61) it must be used for this section.

The gross area is 0.94 square inches, and since one rivet hole will be punched in the section, the net area is 0.73 square inches. The required area to resist tension (S 37) is :

$$\frac{7{,}320}{16{,}000} = 0.458 \text{ square inches,}$$

and the required number of rivets (S 41) is :

$$\frac{7{,}320}{3{,}750} = 1.95$$

$$= \text{say 3 field rivets.}$$

Since the above angle is the smallest allowable by the Specifications, and since the numbers in the middle pannel have smaller stresses than the member just designed, the above section will also be used for the diagonals in the middle pannel.

In the design of this bridge, whenever the stress carried by an angle is in excess of the allowable for the net section of the connected leg, both legs of the angle must be connected to the connection plate by clip angles or otherwise. This requirement is made because tests have shown that angles riveted by one leg develop only about 60 per cent of the strength that is obtained

when both legs are connected.

7. The Pedestal.

Fig. 9 shows the general type and dimensions of the pedestal which will be used at the movable end of this bridge. The end reaction is equal to the vertical component of the stress in the end post, and is :

$$\frac{24}{31.25} \times 73,900 = 56,800 \text{ pounds,}$$

which requires a bearing area on the masonry (§ 41) of :

$$\frac{56,800}{600} = 94.7 \text{ square inches.}$$

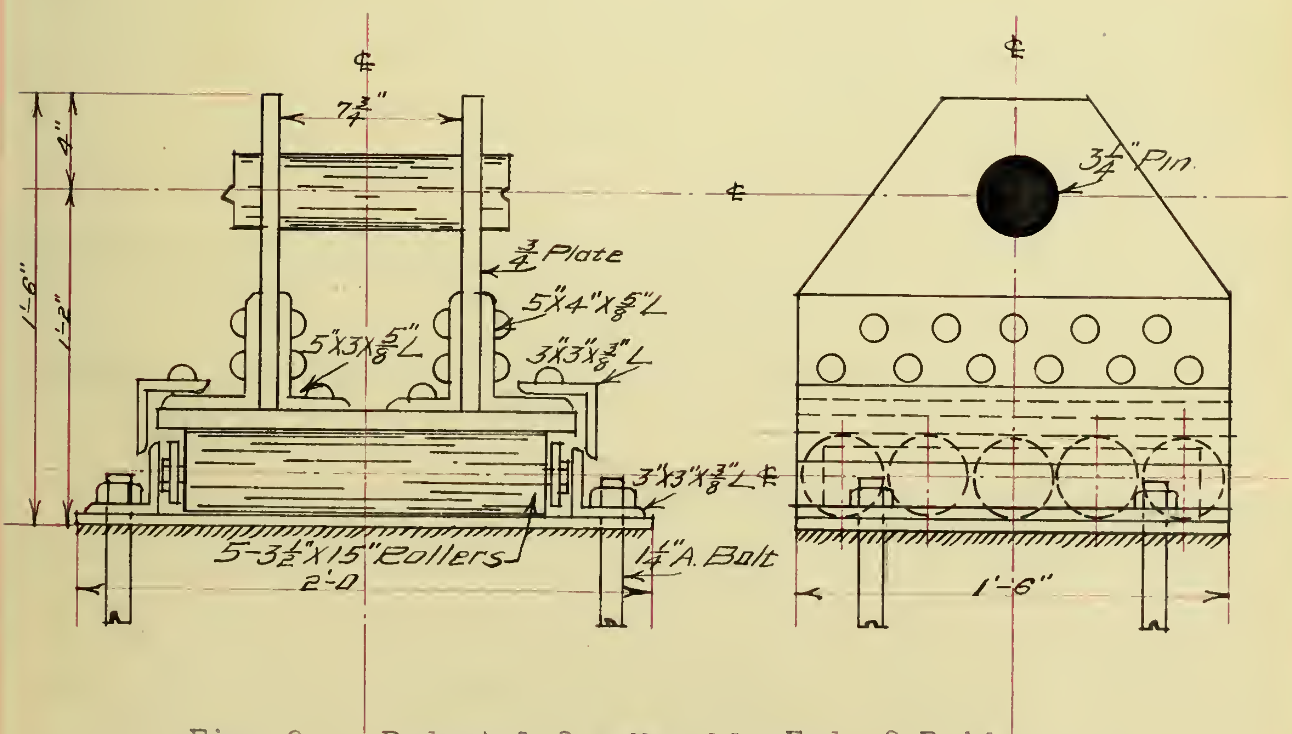


Fig. 9. Pedestal for Movable End of Bridge.

The length of the masonry plate is assumed to be 18 inches, and therefore the required width is :

$$\frac{94.7}{18} = 5.27 \text{ inches.}$$

The total bearing area (S 41) required for one vertical plate is :

$$\frac{56,800}{2 \times 24,000} = 1.19 \text{ square inches,}$$

and since the pin at L_0 is $3\frac{1}{4}$ inches in diameter, the required thickness is :

$$\frac{1.19}{3.25} = 0.365$$

$$= \text{say } 3/4 \text{ inch (S 86) .}$$

These vertical plates will be connected to the bearing plate (S 86) by 5 by 3 by 5/8 inch angles placed on each side of the plates. The masonry plate should extend out beyond the bearing plate 3 inches to allow sufficient room for the anchor bolts, which must be $1\frac{1}{4}$ inches in diameter (S 86). The vertical plates must be so spaced as to allow the end post to be placed between them. A distance of 7 3/4 inches between the vertical plates will allow a clearance of 1/8-inch between these plates and the end post. Since this distance is 7 3/4 inches, the width of the upper bearing plate is found to be 16 inches, and that of the masonry plate 22 inches. This width is much greater than the required width but it must be used.

The distance from the center of the pin to the top of the masonry must be the same at both the roller and the fixed end of the bridge. This distance must be such that the angles of the shoe will clear the bottom chord member. Since the first section of the bottom chord is 6 inches deep, the top of the angles must be at least 3 inches below the center line of the pin. The required distance from the center line of the pin to the base of the angles at the roller end must be 8 inches or more, and from the pin to the

masonry it must be 13 inches or more.

The rivets through the vertical leg of the shoe angles are in double bearing in the 5/8-inch angles, in single bearing in the 3/4-inch vertical plate, and in double shear. Since this connection is weakest in shear, the required number of rivets (S 40, 86) is :

$$\frac{56,800}{2 \times 10,600} = 2.68$$

$$= \text{say 11 shop rivets.}$$

The rivets in the horizontal legs of these angles take no stress and, as is customary, they will be staggered with those of the vertical leg.

The rollers (S 84) shall be $3\frac{1}{2}$ inches in diameter, and the unit stress (S 41) per linear inch is :

$$3.5 \times 600 = 2,100 \text{ pounds,}$$

which requires:

$$\frac{56,800}{2,100} = 27 \text{ linear inches.}$$

Since the width of the upper bearing plate is 16 inches, and the length is 18 inches, five rollers may be used, each having a length of 15 inches, making a total of 75 linear inches. These rollers will be connected by one top bolt passing through the 2 by $\frac{1}{4}$ inch guide plate into the roller. The guide angles will be 3 by 3 by 3/8-inch.

The pedestal at the fixed end is of the same general design as that shown above. The principal difference being the omission of the rollers, and hence the shoe angles will be connected directly to the masonry plate. This will, of course, necessitate the lengthening of the vertical plates in order that the pin may

be at the same distance or height above the masonry at the fixed end as at the roller end.

8. Fence.

The stresses likely to come in the fence can not be accurately determined and therefore we must resort to what is considered good engineering practice. The fence will be composed of two 4" x 4" x 3/8" angles spaced 2 feet apart vertically. They will be riveted to the intermediate posts, and will be connected together by 2" x ¼" lacing bars.

9. Abutments.

It is seen on Plate I that the base of the abutments may be satisfactorily located at elevation 91.0, at which point they will have a good bearing on firm clay soil. Since the top of the abutments must be at the same elevation as that of the pier, the height is 10 feet, and according to Cooper's Standards (K 299) the dimensions are as shown in Fig. 10.

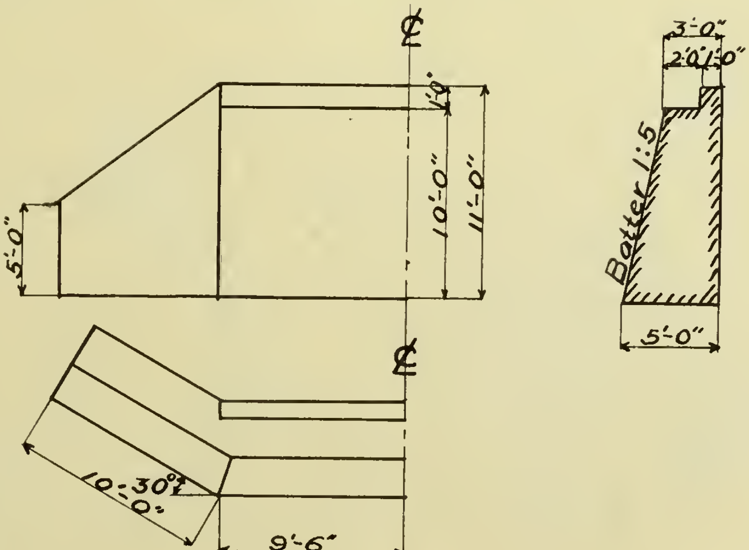


Fig. 10. General Dimensions of Abutment.

The abutments are of Portland cement concrete having the following proportions:

Cement	1 part
Sand	$2\frac{1}{2}$ parts
Broken Stone	5 parts

The cubical contents of each abutment is 50 cubic yards, and therefore the required amount of concrete for the two abutments is 100 cubic yards. The price of the concrete per cubic yard in place is assumed to be $8.00, the same price as was assumed for the concrete used in the pier, and therefore the total cost of the two abutments is $800.00.

IV. COST.

A detailed estimate of the cost will now be made of the above designed bridge. The cost of the steel will be estimated at the mill at Pittsburg, Pa.

Owing to the fact that the present wooden structure, on the site of the proposed bridge may be used as a sub-structure, no allowance need be made for the expense of false work for the erection of the bridge.

The bridge is composed of I-beams, channels, angles, bars, plates, and pin rounds. Table II gives the total weight of the above material for one span.

Table II.

Weight of Steel in One Span of Bridge Exclusive of Joists, End Struts and Fence .

	Weight Pounds	Percent of Total Weight.	Cost per Pound, Cents.	Percentage of Pound Cost, Cents.
Beams and Channels	20430	46	1.70	0.770
Angles	6960	16	1.70	0.272
Bars	10380	24	1.60	0.390
Plates	5540	13	1.60	0.208
Pin Rounds	490	1	2.00	.020
	43800	100		1.66

The average cost of steel at mill - - - - -	1.66	cents per pound
Waste in fabrication, 4 per cent - - - - -	0.066	" " "
Paint material - - - - - - - - - - - - - - -	0.010	" " "
Freight, Pittsburg to Dunloith, Miss. - - -	0.180	" " "
Average Cost of Steel at Mill - - -	1.916	cents per pound.

The joists, end struts and hub guard or fence will take the rate of 1.70 cents per pound at the mill:-

The average cost of steel at mill - - - - -	1.700	cents per pound
Waste in fabrication, 2 per cent - - - - - -	0.034	" " "
Paint material - - - - - - - - - - - - - - - -	0.010	" " "
Freight, Pittsburg to Dunleith - - - - - - -	0.180	" " "
Average cost of steel at the shop - -	1.924	cents per pound.

Shop Cost of Steel in Bridge, Exclusive of joists, fence, etc.:-

Average cost of steel at shop - - - - - - -	1.916	cents per pound
Shop cost, including drafting - - - - - - -	0.750	" " "
Total shop cost - - - -	2.666	" " "
Freight, Pittsburg to Dunleith - - - - - - -	0.180	" " "
Total cost at Dunleith - - - - - - - -	2.846	cents per pound.

Shop Costs of Joists, Fence and End Struts:-

Average cost of steel at the shop - - - - -	1.916	cents per pound
Shop cost, including drafting - - - - - - -	0.250	" " "
Total shop cost - - - - - - - - - -	2.166	" " "
Freight, Shop to Dunleith - - - - - - - - - -	0.180	" " "
Total cost at Dunleith - - - - - -	2.346	cents per pound.

Erection :-

Hauling 38 tons 2 miles at 25 cents per ton for and 50 cents per ton for loading - - - - - - - -	$ 28.50
Labor for erecting, 30 days, labor at $3.00 - - - - - -	90.00
Transportation of men and tools - - - - - - - - - - - - -	60.00
Labor, painting bridge 2 coats, 10 days at $3.00 - - - -	30.00
Labor, erecting floor lumber 11,000 ft. B.M. at $3.00 - -	48.00
Total cost of erection - - - - - - - - -	$256.50

Summary of Cost of Superstructure.

Steel, 43,800 lb. at 2.846 cents per pound - - - - - -	$ 1248.30
Joists, fence, etc. 21,700 lb. at 2.346 - - - - - - - -	509.05
Lumber, 11,000 ft. B.M. at $30.00 - - - - - - - - - - -	330.00
Paint, 20 gallons at $1.25 - - - - - - - - - - - - - - -	25.00
Spikes for the floor 400 lb. at $0.03 - - - - - - - - - -	12.00
Cost of Erection -	256.50
Total cost for one span - - - - -	$2381.70.

Two spans at \$2,381.70 - - - - - - - - - - - - - - - -	\$4,763.40
One pier at -	1,592.00
Two abutments -	800.00
Profit 15 per cent - - - - - - - - - - - - - - - - - - -	360.00
Total cost of bridge complete - - -	\$7,576.00